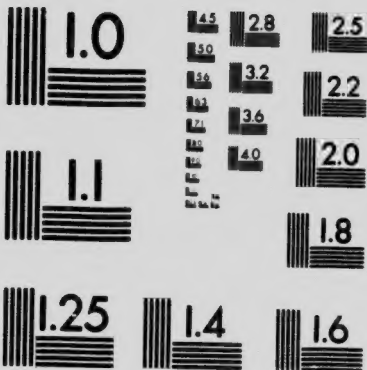




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Structural Problems



Part I



C. R. YOUNG

RB34101

Structural Problems

Part I

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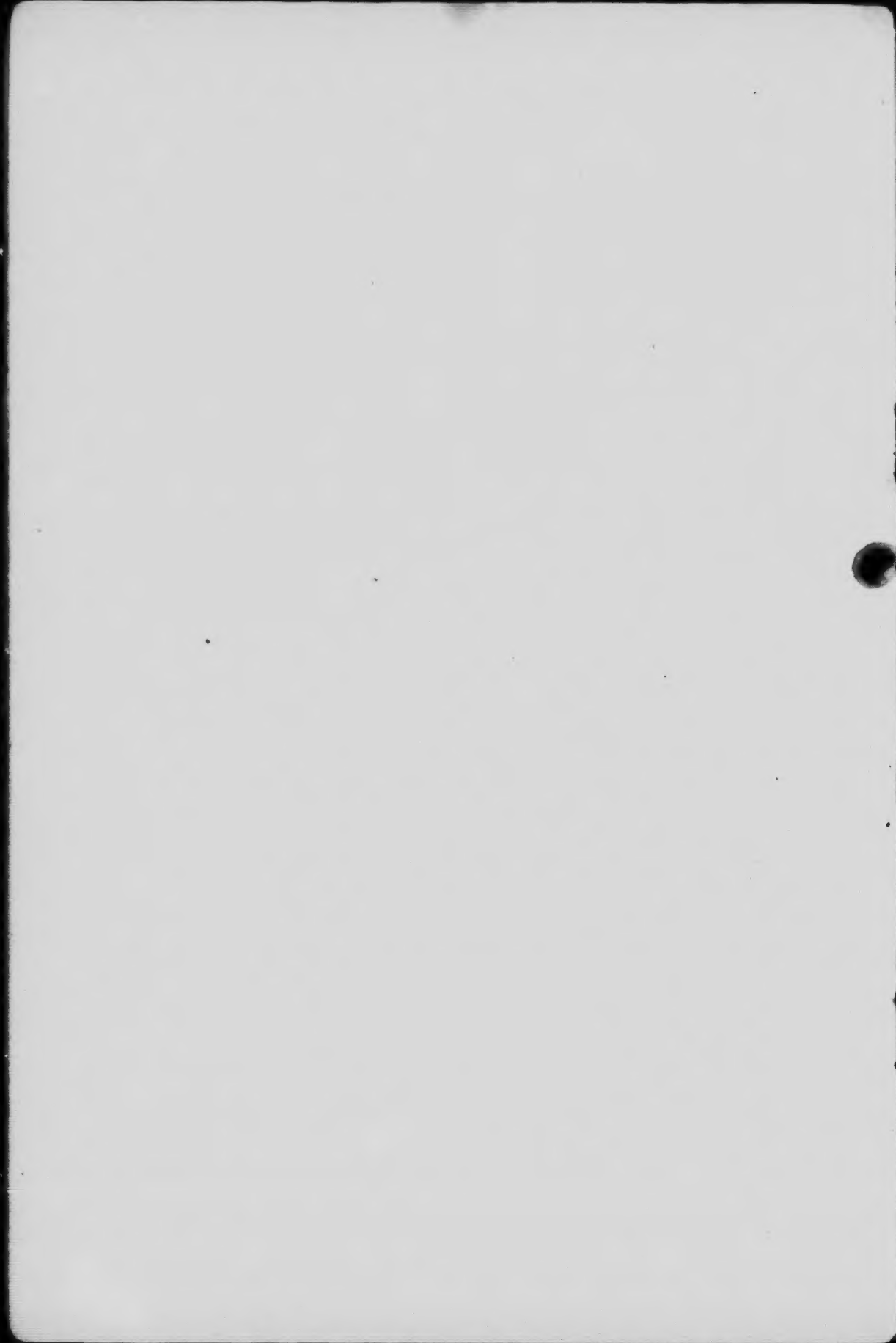
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By C. R. Young

FOREWORD

The typical structural problems herein presented have been prepared for the assistance of students in Elementary Structural Design. While intended primarily for the author's classes, they may be found of use to others who are striving to improve themselves in the subject. Space has not permitted lengthy explanation or derivation of formulae, and hence the booklet has been fashioned to serve merely as a supplement to lectures and standard text books and not in any sense as a treatise.

C. R. Y.

University of Toronto,
Toronto, September, 1920.



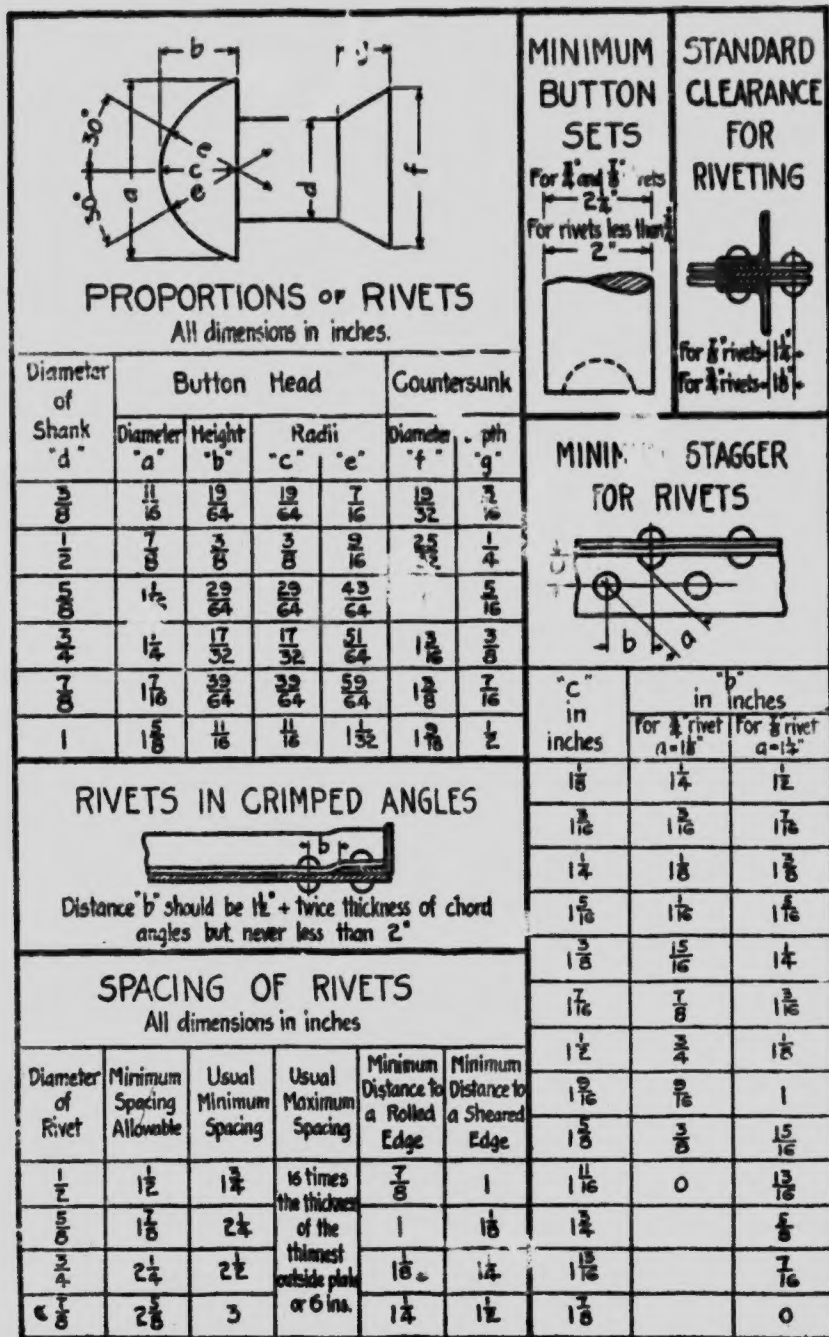


FIG. 1.—RIVETS AND RIVETING STANDARDS

GAUGES OF ANGLES

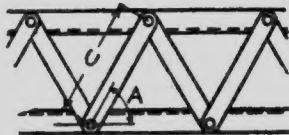
All dimensions in inches.



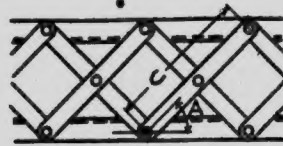
Leg	Gauge g	Maximum Rivet	Leg	Gauge g	Maximum Rivet	Leg	Gauge g ₁	Gauge g ₂	Maximum Rivet
$\frac{3}{4}$	$\frac{7}{16}$	$\frac{1}{4}$	$2\frac{1}{2}$	$\frac{13}{8}$	$\frac{3}{4}$	5	2	$1\frac{1}{2}$	$\frac{7}{8}$
$\frac{7}{8}$	$\frac{1}{2}$	$\frac{1}{4}$	$2\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{4}$	6	$2\frac{1}{2}$	$2\frac{1}{2}$	$\frac{7}{8}$
1	$\frac{5}{8}$	$\frac{1}{4}$	3	$1\frac{1}{2}$	$\frac{7}{8}$	7	$2\frac{1}{2}$	3	$\frac{7}{8}$
$1\frac{1}{8}$	$\frac{11}{16}$	$\frac{3}{8}$	$3\frac{1}{2}$	2	$\frac{7}{8}$	8	3	3	$\frac{7}{8}$
$1\frac{1}{4}$	$\frac{3}{4}$	$\frac{3}{8}$	4	$2\frac{1}{2}$	$\frac{7}{8}$	When the thickness of angles is such that the distance "A" exceeds the dimensions given below, the gauges g $\frac{1}{4}$" for $1\frac{1}{2}$" and 2" legs. $\frac{1}{8}$" - legs greater than 2" or g, must be increased or smaller rivets than the maximum used.			
$1\frac{3}{8}$	$\frac{7}{8}$	$\frac{3}{8}$	$4\frac{1}{2}$	$2\frac{3}{4}$	$\frac{7}{8}$				
$1\frac{1}{2}$	$\frac{7}{8}$	$\frac{3}{8}$	5	3	$\frac{7}{8}$				
$1\frac{3}{4}$	1	$\frac{1}{2}$	6	$3\frac{1}{2}$	$\frac{7}{8}$				
2	$1\frac{1}{8}$	$\frac{5}{8}$	7	4	$\frac{7}{8}$				
$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{5}{8}$	8	$4\frac{1}{2}$	$\frac{7}{8}$				

FIG. 2.—GAUGES OF ANGLES

LACING

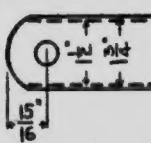


Angle A = 60° about
The thickness of the lattice bars should not be less than $\frac{5}{16}$

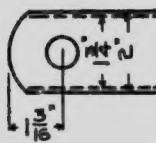


Angle A = 45° about
The thickness of the lattice bars should not be less than $\frac{5}{16}$

$\frac{1}{2}$ " rivet



$\frac{5}{8}$ " rivet



$\frac{3}{4}$ " rivet



$\frac{7}{8}$ " rivet



$\frac{3}{4}$ " for $\frac{1}{2}$ " rivets
 $\frac{5}{8}$ " for $\frac{3}{4}$ " rivets
 $\frac{1}{2}$ " for $\frac{1}{2}$ " and $\frac{5}{8}$ " rivets

MANNER OF CUTTING
AND PUNCHING OF BARS

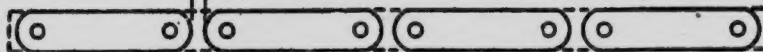


FIG. 3.—LACING STANDARDS OF THE AMERICAN BRIDGE CO.

DETERMINATION OF TRUE NET SECTION OF TENSION MEMBERS

The theoretically correct number of rivet holes to be deducted from the gross right sectional area of a tension member (see sketch in upper right hand corner of Fig. 4, 5 or 6) to give the true effective section is:

$$n = 1 + x_1 + x_2 + x_3 + \dots \text{etc.} \quad (1)$$

$$\text{where } x = \frac{g}{h} - \frac{2(g^2 + s^2 - h\sqrt{g^2 + s^2})}{h^2 + \sqrt{g^2 + 4s^2}}$$

g = distance between gauge lines,

h = diameter of rivet hole (dia. of rivet + $\frac{1}{8}$ in.),

s = stagger of rivet with respect to previous rivet.

Formula (1) is to be applied to alternative sections, the successive terms representing the deductions for successive holes considered in a chain across the member. The particular group of rivets to be considered is that which will give the greatest total deduction, whether the rivets lie in adjacent gauge lines or not.

To obviate the large amount of work required in solving formula (1) for each case, the diagrams given in Figures 4, 5 and 6 have been prepared. The curves give the exact deductions for the three commonest sizes of rivets in structural work, viz. $\frac{5}{8}$, $\frac{3}{4}$, and $\frac{7}{8}$ in.

Various approximate rules have been formulated for use where no diagrams are available. D. B. Steinman's second rule is:

"After deducting the first hole, each successive hole is to be deducted if its stagger s with respect to the preceding hole does not exceed one-half the gauge g , and no deduction is to be made if the stagger equals or exceeds the gauge. For intermediate values of the stagger ratio, a fraction of the hole is to be deducted as may be obtained by interpolation, that is, deduct $2(g-s)/g$."

In order to conform more closely than the above to the requirements dictated by both theory and experiment, without applying the cumbersome formula (1), or using diagrams, the author has proposed that deductions be calculated in each component part of the member for the maximum number of rivet holes obtained by applying to alternative sections the approximate summation,

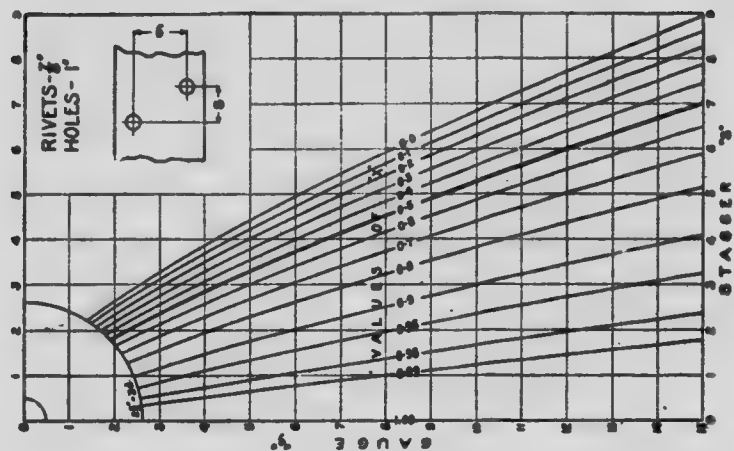


FIG. 6

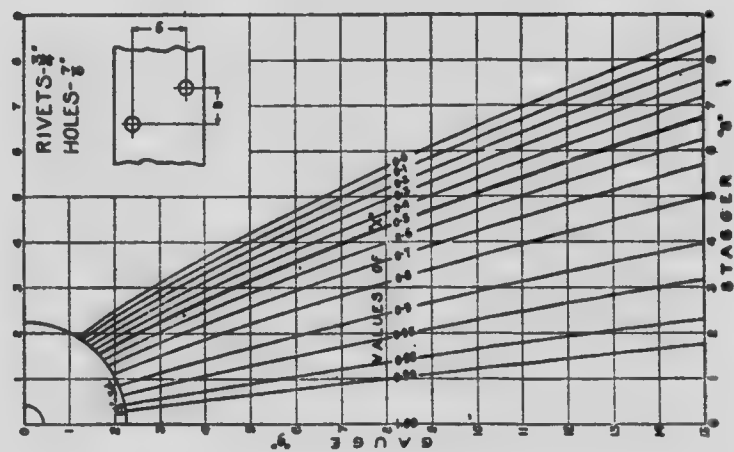


FIG. 5

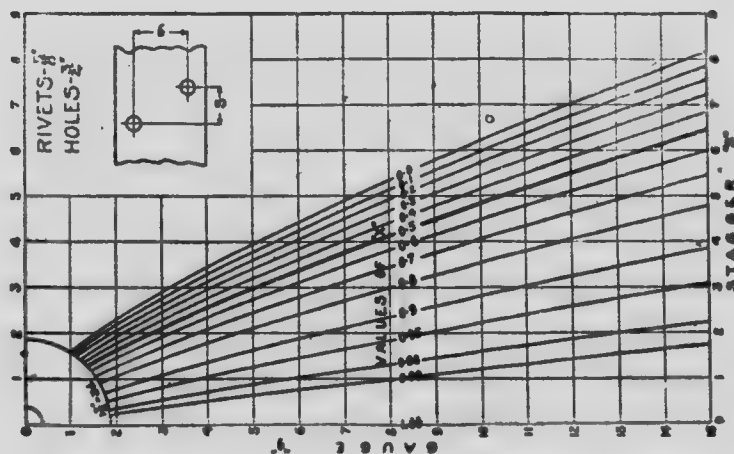


FIG. 4

THEORETICALLY CORRECT DEDUCTIONS OF RIVET HOLES FOR VARIOUS GAUGES AND STAGGERS

$$1 + 1.50 \left(1 - 0.8 \frac{s_1}{g_1}\right) + 1.50 \left(1 - 0.8 \frac{s_2}{g_2}\right) + \text{etc.} \quad (2)$$

to n terms.

The successive terms represent the deductions for successive holes considered; g_1, g_2 , etc., represent distances between successive lines of rivets; s_1, s_2 , etc., represent staggers of rivets with respect to the previous rivet. A term $1.50 (1 - 0.8 s/g)$ is to be considered as never having a value in excess of unity, since obviously the deduction could not be more than one hole for each rivet.

DESIGN OF CONNECTION FOR SINGLE-ANGLE TENSION MEMBER

DATA

Section of Member. One angle, $4 \times 3 \times \frac{3}{8}$ in.

Form of Detail. Long leg riveted directly to gusset; 3-in. leg connected by lug angle having outer end flush with outer end of member; detail to be of maximum efficiency, that is such that only one rivet hole need be deducted; gusset plate $\frac{3}{8}$ in.; arrangement as in Fig. 7.

Load. Safe capacity of member.

Permissible Stresses.

Tension = 16,000 lb. per sq. in.,

Shearing on shop rivets = 12,000 lb. per sq. in.,

Bearing on shop rivets = 24,000 lb. per sq. in.

Rivets. $\frac{7}{8}$ in. dia., power-driven, shop.

Assumptions. Consider full net section as effective; diameter of rivet hole = $\frac{7}{8} + \frac{1}{8} = 1$ in.;

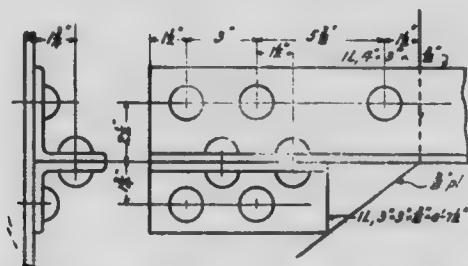


FIG. 7—CONNECTION OF SINGLE ANGLE FOR MAXIMUM EFFICIENCY

NUMBER OF RIVETS

Assuming that detail is such as to require the deduction of only one rivet hole, net area of $4 \times 3 \times \frac{3}{8}$ -in. angle = $2.48 - 1 \times 0.375 = 2.10$ sq. in. At 16,000 lb. per sq. in., the capacity = $2.10 \times 16,000 = 33,600$ lb.

Least value of one rivet is single shearing value = $0.60 \times 12,000 = 7,200$ lb.

Number of rivets required = $33,600 / 7,200 = 4.7$. Use 5 rivets.

DISTRIBUTION OF RIVETS

To ensure equalized stress over the section of the angle, the five rivets should be divided between the two legs of the main angle in proportion to their gross areas. The 4-in. leg should therefore contain $5 \times (4 \times 0.375) / \{(4 + 2.625) \times 0.375\} = 3.0$ rivets. Three rivets will therefore be used to connect the 4-in. leg directly to the gusset and two rivets to connect the 3-in. outstanding leg to the lug angle.

In order to keep the gusset plate as small as practicable and the lug angle short, 3-in. spacing, the normal minimum for $\frac{7}{8}$ -in. rivets, will be used. Two rivets, spaced 3-in. are first located in the 4-in. leg of the main angle, with the outer one $1\frac{1}{2}$ in. from the end—the normal distance of a $\frac{7}{8}$ -in. rivet from a sheared edge. Two rivets are then placed directly opposite these through the lug and the gusset. The necessary two rivets in the outstanding leg of the lug angle to develop the stress borne by the rivets in the other leg are spaced 3 in. apart and staggered $1\frac{1}{2}$ in. with those in the other leg; to facilitate driving.

For a connection of maximum efficiency, the remaining, or first, rivet in the 4-in. leg must be located at such a distance forward of the first rivet in the outstanding leg that only one hole need be deducted. The distance apart of the gauge lines, if the angle were developed, assuming a gauge of $2\frac{1}{2}$ in. in the 4-in. leg and $1\frac{3}{4}$ in. in the 3-in. leg would be $2.50 + 1.75 - 0.375 = 3.875$ in. The necessary stagger of these rivets, in order that only one hole need be deducted, is found from Fig. 6 to be $3\frac{7}{8}$ in. The final spacing is therefore as shown in Fig. 7, there being $5\frac{3}{8}$ in. between the first two rivets in the main angle.

LUG ANGLE

Since the lug angle must transmit to the gusset the proportion

of the total stress that should be borne by the outstanding leg of the main angle, the outstanding leg of the lug angle should be of the same section as the outstanding leg of the main angle, and is preferably of the same width and thickness. The leg in contact with the gusset should be made as narrow as the driving of the rivets in it at the required stagger will permit. In the present case, this is 3 in. The length of the lug is determined by the rivet spacing adopted and by the minimum end distances permissible.

Frequently, the maximum efficiency obtained by making the distance between the first and second rivets greater than that between the other rivets is sacrificed in order to reduce the size of the gusset plate or shorten the main angle.

DESIGN OF A COLUMN

DATA

Type. Bethlehem H-column with web in plane of eccentricity; continuous and of same section for two stories. See Fig. 8.

Loads. From roof 26,000 lb., centrally applied; at first floor down from roof, 20,000 lb. centric load, and 25,000 lb. applied at $2\frac{1}{4}$ in. from inner face of column.

Permissible Stresses, Etc.

Compression, for ratios of l/r up to 125, $= p = 16,000 - 70 \frac{l}{r}$, with a maximum of 13,000 lb. per sq. in., where l = unsupported length in inches and r = least radius of gyration.

Compression, for ratios of l/r between 125 and 200, $= p = 14,750 - 60 \frac{l}{r}$.

$E = 29,000,000$ lb. per sq. in.

SECOND STORY FROM ROOF

Load on this tier = 26,000 lb. from roof + 20,000 lb. centric load at floor "Y" + 25,000 lb. eccentric load at floor "Y" + weight of column for two stories, say, 1,000 lb., or a total of 72,000 lb.

Unsupported length = approximately 13 ft. = $13 \times 12 = 156$ in. Assume an 8-in. Bethlehem H-column at 34.5 lb. per ft.; area = 10.17 sq. in.; radius of gyration about the axis 1-1 (Fig. 8) = 3.46 in.; radius of gyration about the axis 2-2 = 2.01 in. Hence, l/r in plane of web = $156/3.46 = 45$ and at right angles to plane of web, it = $156/2.01 = 78$.

Maximum permissible stress, considering strength in plane of web, therefore = $p = 16,000 - 70 \times 45 = 12,800$ lb. per sq. in., and at right angles to plane of web, it = $p = 16,000 - 70 \times 78 = 10,500$ lb. per sq. in.

Consider strength in plane of web. Area required = $P/p + M_1 y_1 / r^2 p$, where P = total vertical load (including eccentric load), p = maximum permissible stress in lb. per sq. in., M_1 = moment due to eccentricity, y_1 = distance from centre of gravity of section to extreme fibre, r = radius of gyration in plane of web. Substituting numerical values, $A = 72,000/12,800 + (25,000 \times 6.25 \times 4) / (3.46)^2 \times 12,800 = 5.63 + 4.07 = 9.70$ sq. in. As the area provided is 10.17 sq. in., the section is adequate.

Consider strength at right angles to plane of web. Eccentricity does not apply in this direction, hence load is merely a centric load of 72,000 lb. Area required = $72,000/10,500 = 6.86$ sq. in. Section is therefore strong enough in this direction.

ALLOWANCE FOR DEFLECTION

Checking the design to allow for effect of the total axial load acting on the deflection in the column arising from eccentricity of loading, the maximum resultant stress on the extreme fibre on the inner face of the column = $f_1 + f_2 = P/A + M_1 y_1 / (I - Pl^2/10E)$, where f_1 = stress due to total axial load, considering it as centric; f_2 = stress due to eccentricity; A = area of section; I = moment of inertia of section in plane of web; E = modulus of elasticity. Substituting numerical values, $f_1 + f_2 = 72,000/10.17 + (25,000 \times 6.25 \times 4) / \{121.5 - 72,000 \times (156)^2 / (10 \times 29,000,000)\} = 7,100 + 5,400 = 12,500$ lb. per sq. in.

As this is below the permissible stress, the section is adequate.

DESIGN OF A POST JIB CRANE

DATA

Type. Post, or mast, and boom, with boom supported by overhead tie, as shown in Fig. 9.

Load. Two tons on trolley, anywhere from mast to end stop of boom.

Permissible Stresses.

Tension on net section of tie rods, = 15,000 lb. per sq. in., Combined compression and bending on mast or boom = $p = 19,000 - 100 l/r$, with a maximum of 13,000, where l = unsupported length in inches and r = least radius of gyration in inches.

Compression on flange of beam = $19,000 - 300 l/b$, where l = unsupported lateral length, and b = breadth of flange.

Impact. 25% of the live load stresses.

Rivets and bolts. $\frac{3}{4}$ in. dia.

Assumptions. For computation of stresses, assume mast as 16 ft. high and span of boom from mast to tie rod connection as 8 ft. Minimum depth of beam for boom to accommodate standard overhead trolley = 8 in.

OVERHEAD TIE

Maximum live load concentration at connection of tie to boom = 4,000 lb. Impact allowance = $0.25 \times 4,000 = 1,000$ lb. Hence, neglecting dead load, total concentration = 5,000 lb. Since slope of tie is $4\frac{1}{2}$ in. 12, stress in tie = $5,000 \times \{(4.5)^2 + (12)^2\}^{1/2} / 4.5 = 14,200$ lb.

Required net area of tie = $14,200 / 15,000 = 0.95$ sq. in. Use two 1-in. dia. rods, not upset, for which area = $2 \times 0.55 = 1.10$ sq. in. at root of thread.

BOOM

Maximum live load moment in 8-ft. segment = $Wl/4 = 4,000 \times 96/4 = 96,000$ in.-lb. For the overhanging 3-ft. segment, assuming that centre of gravity of the load at its extreme outer position is 2 ft. from the tie connection, the moment = $Wl = 4,000 \times 24 = 96,000$ in.-lb., that is equal to moment in 8-ft. segment, but of opposite sign. This is the ideal arrangement, so far as moment is concerned. Adding 25% impact, the moment = 120,000 in.-lb.

Horizontal thrust, P , applied by tie approximately 6 in. above centre of gravity of boom when trolley is in position to produce maximum moment in the 8-ft. segment, that is at its centre, = $2,500 \times 12/4.5 = 6,700$ lb., including impact.

The total maximum stress in the boom on the upper fibre of the 8-ft. segment at its centre = stress due to cross bending + stress due to eccentric thrust from tie.

Stress due to cross bending, $f = M/S$, where M = moment and S = section modulus. Assuming an 8-in. I at 18.4 lb., $f = 120,000/14.2 = 8,500$ lb. per sq. in.

Stress due to P , including effect of eccentricity, approximately $= f_1 + f_2 = P/A + My_1/I$, where A = area of section, M = moment due to eccentricity, y_1 = distance from centre of gravity of section to extreme fibre, and I = moment of inertia. This $= 6,700/5.33 + (6,700 \times 6 \times 4)/56.9 = 1260 + 2840 = 4,100$ lb.

Total maximum stress $= 8,500 + 4,100 = 12,600$ lb. per sq. in. Safe stress in vertical plane $= 13,000$ lb. per sq. in.; since $l/r = 96/3.27 = 30$. Safe stress in horizontal plane $= 19,000 - 300 l/b = 19,000 - 300 \times 90/4 = 12,250$ lb. per sq. in. Hence boom is slightly overstressed due to lateral buckling, but not enough to require revision.

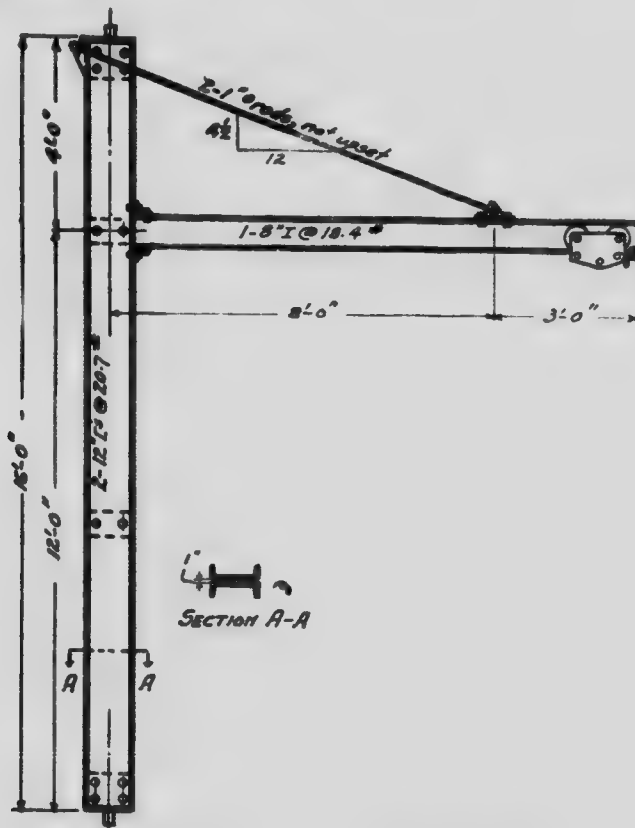


FIG. 9—DESIGN OF A POST JIB CRANE

MAST

Vertical load at top, due to maximum pull of tie, = 5,000 lb., including impact but neglecting dead load. This is applied at approximately 8 in. from centre of column on opposite side to boom.

Horizontal thrust on mast, with trolley at tie connection to boom = $5,000 \times 12/4.5 = 13,400$ lb., including impact. Horizontal reaction at top of mast due to boom thrust = $13,400 \times 12/16 = 10,100$ lb.

Moment at boom connection due to lateral thrust = $10,100 \times 48 = 485,000$ in.-lb. Moment due to eccentric connection of tie to top of mast = $5,000 \times 8 = 40,000$ in.-lb., of opposite sense. Net moment = $485,000 - 40,000 = 445,000$ in.-lb.

Assuming mast to be of two 12-in., channels at 20.7 lb., maximum stress at boom connection = $f_1 + f_2 = P/A + My_1/I = 5000/12.06 + 445,000 \times 6/256.2 = 400 + 10,400 = 10,800$ lb. per sq. in.

Maximum permissible stress due to columnar action in plane of webs, with $l/r = 192/4.41 = 42$ is 13,000 lb. per sq. in. Considering lateral buckling effect, is $19,000 - 300 l/b = 19,000 - 300 \times 192/6.9 = 10,750$ lb. per sq. in., or almost exactly the computed maximum stress.

DETAILS

Head and foot castings are designed with a 1-in. web to be bolted or riveted between the channels. A casting, preferably of steel, connects the tie to the boom.

DESIGN OF A RIVETED STEEL COLUMN BASE

DATA

Type. Riveted steel plate and angle base for a 10-in. Bethlehem H-column at 49 lb. per ft., of the type shown in Fig. 10.

Load. Vertical downward load of 170,000 lb. Wind force inconsiderable.

Permissible Stresses

Bending = 16,000 lb. per sq. in.,

Bearing on end of column = 16,000 lb. per sq. in.,

Shearing on shop rivets = 12,000 lb. per sq. in.,

Bearing on shop rivets = 24,000 lb. per sq. in.,

Bearing on concrete = 500 lb. per sq. in.

Rivets. $\frac{3}{4}$ in. dia. Anchor bolt holes $\frac{1}{8}$ in. larger than anchor bolts.

Assumptions. Consider 40% of total axial load as carried directly to the masonry by bearing of the faced end of the column shaft.

BASE PLATE

Required area of base plate = total load $\div$ permissible pressure on masonry = $170,000/500 = 340$ sq. in. Thickness of such a plate is determined by judgment and experience in accordance with the size and shape of the column shaft. For the present column it will be fixed at $\frac{3}{4}$ in. To satisfy requirements of loading, and to facilitate details, use a base plate $18 \times \frac{3}{4} \times 1$ ft. 7 in., giving an area of 342 sq. in. The dimension parallel to the web is fixed at 18 in. to accommodate two base angles with $3\frac{1}{2}$ -in. horizontal legs and two $\frac{3}{8}$ -in. side plates. The 19-in. dimension necessarily follows.

SIDE PLATES AND BASE ANGLES

Since only 40% of total axial load is assumed to be transferred to masonry by faced end bearing of column shaft, the remaining 60%, or $170,000 \times 0.60 = 102,000$ lb., must be delivered to base plate by side plates and base angles. To make this possible, enough rivets must be placed through column shaft to develop 102,000 lb. Assume 16 rivets through the flanges, for which the least value (single shear) is $0.44 \times 12,000 = 5280$ lb., and 4 rivets through the web, for which the least value (bearing on 0.36-in. web) is $0.36 \times 0.75 \times 24,000 = 6480$ lb. The total safe resistance of these two groups of rivets, therefore, = $16 \times 5280 + 4 \times 6480 = 110,500$ lb., which is adequate for the load.

The side plates, which for a column of the section considered should be about $\frac{3}{8}$ in. thick, will extend across the full width of the base plate to help transfer load out to its edges, and will be 12 in. deep so as to accommodate two rows of rivets outside the base angles, which are riveted to it.

The base angles riveted to the column flanges are run full width of the base and two rivets are driven through each angle into the side plate. There are, therefore, 6 rivets in single shear through each angle, so that the angles will deliver to the base plate $12 \times$

5280 = 63,400 lb., or 37% of the total column load, leaving 23% to be delivered by the side plates and the base angles on the web.

Four rivets through the base angles in the column web will develop $4 \times 6480 = 25,900$ lb., or 15% of the total column load. This leaves only 8% of the total column load to be delivered to the base plate by the two side plates.

All vertical rivets through base angles must be countersunk on the under side. Only sufficient rivets are employed to hold the angles and plate tightly together.

Anchor bolt holes are provided $\frac{1}{8}$ in. larger than the anchors, as is customary.

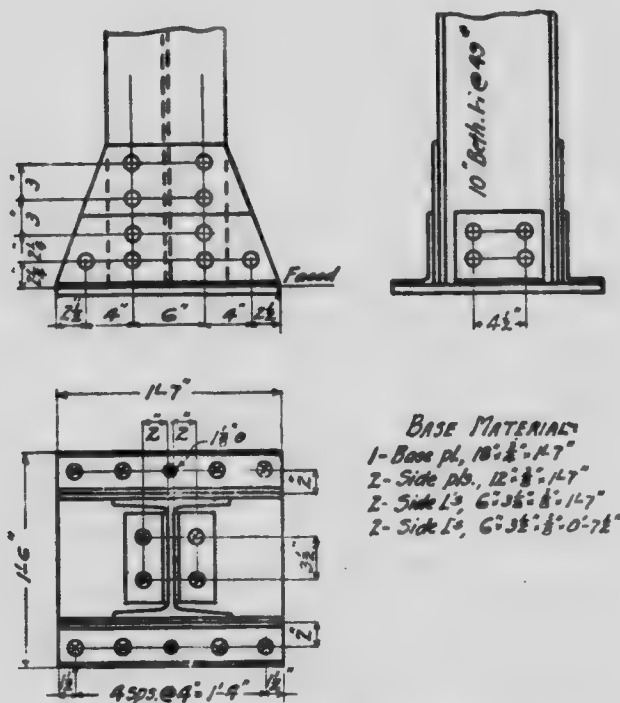


FIG. 10—TYPICAL RIVETED BASE FOR H-COLUMN

DESIGN OF COLUMN BRACKET "A"

DATA

Type. Bracket of type shown in Fig. 11.

Load. 40,000 lb. applied $2\frac{1}{4}$ in. from face of column.

Permissible Stresses.

Shearing on shop rivets = 10,000 lb. per sq. in.,

Bearing on shop rivets = 20,000 lb. per sq. in.,

Tension on rivet shafts = 7,000 lb. per sq. in.

Rivets. $\frac{3}{4}$ in. dia.

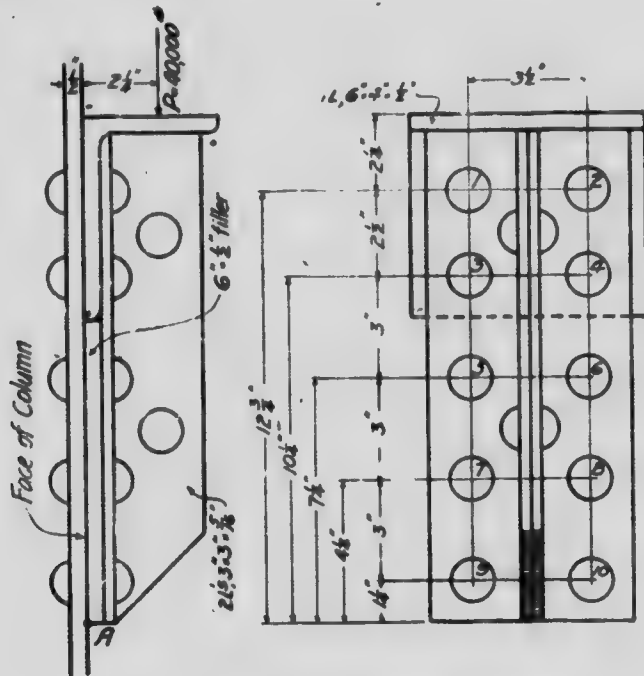


FIG. 11—DESIGN OF A COLUMN BRACKET

SOLUTION

Adopt as bracket material a $6 \times 4 \times \frac{1}{2}$ -in. shelf angle and two $3 \times 3 \times \frac{5}{16}$ -in. stiffener angles with a $\frac{1}{2}$ -in. filler underneath.

Number of rivets necessary to transmit load in single shear = $40,000/4,420 = 9 +$, say 10. In bearing on $\frac{5}{16}$ -in. stiffener angles, number would be less. Assume 10, and investigate capacity to resist turning about point A.

Moment about point A = $40,000 \times 2.25 = 90,000$ in.-lb.

Tensile stress on any rivet

$$= T_n = Md_n / \Sigma d^2$$

$$\Sigma d^2 = 2 \{ (1.25)^2 + (4.25)^2 + (7.25)^2 + (10.25)^2 + (12.75)^2 \}$$

$$= 679.2.$$

For the most seriously stressed rivets, 1 and 2, $d_n = 12.75$.

$$\text{Hence } T_1 = T_2 = 90,000 \times 12.75 / 679.2 = 1690 \text{ lb.}$$

Safe tension on rivet = $.442 \times 7,000 = 3,094$ lb. and bracket is therefore sufficient. Number of rivets cannot be reduced without exceeding safe shearing stress on rivets.

DESIGN OF COLUMN BRACKET "B"

DATA

Type. Riveted steel bracket of highly eccentric type, shown in Fig. 12, connected to $\frac{3}{8}$ -in. column flange.

Load. Vertical downward load of 24,000 lb. applied at 9 in. from column face, by an 18-in. I at 54.7 lb. at right angles to bracket.

Permissible Stresses.

Shearing on shop rivets = 10,000 lb. per sq. in.,

Bearing on shop rivets = 20,000 lb. per sq. in.,

Tension on rivet shafts = 7,000 lb. per sq. in.

Rivets. $\frac{3}{4}$ in. dia.

Assumptions. Assume that the applied load does not bear directly on the upper edge of the web plate and that the vertical edge of the plate does not bear against the side of the column.

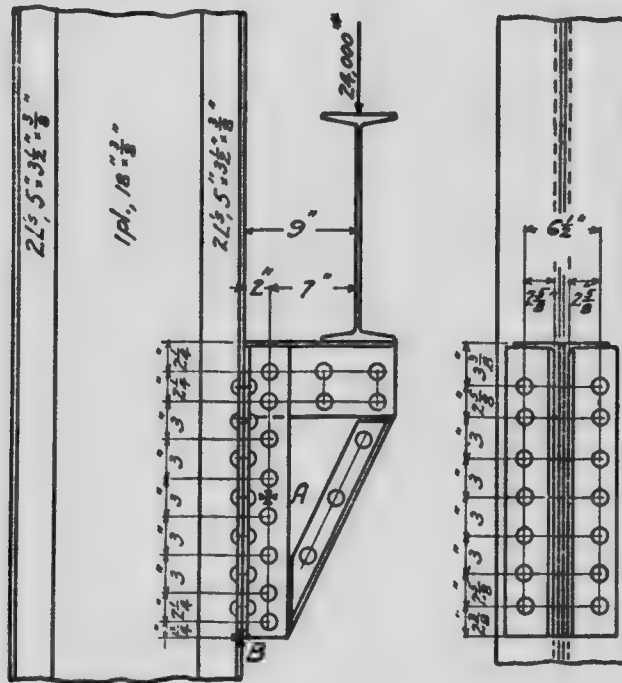
BRACKET MATERIAL

To ensure lateral stiffness and to give reasonable bearing values for rivets through it, assume the web plate as $\frac{3}{8}$ in. Size of this plate will be determined by necessary length of horizontal seat angles to accommodate beam and depth of vertical angles required to give an adequate number of rivets for connection with the column. The seat angles will be $6 \times 3\frac{1}{2} \times \frac{3}{8}$ in., with 6-in. legs vertical, so as to provide space for the necessary number of rivets through the web plate. For the vertical connection angles, use two $4 \times 3\frac{1}{2} \times \frac{3}{8}$ -in. angles, with 4-in. legs against the column to suit the material

of the column flange. If ratio of the length of the unsupported outer edge of the web plate to its thickness is over 30, stiffener angles will be used.

RIVETING OF SEAT ANGLES

Neglecting the bearing of the load on the edge of the web plate, enough rivets must be put through the seat angles to transfer 24,000 lb. to the $\frac{3}{8}$ -in. web. Since the least value of a rivet (bearing value) = $\frac{3}{4} \times \frac{3}{8} \times 20,000 = 5,630$ lb., this necessitates $24,000/5,630 = 4.3$ rivets. Put 6 rivets through the seat angles, 4 of them outside the vertical connection angles. Part of the load will be transferred by the stiffness of the seat angles to the vertical angles through bearing of the seat angles on them and by the two rivets directly connecting them to the vertical angles. The spacing of the rivets in the seat angles, and the length of the latter will be as shown in Fig. 12.



BRACKET MATERIAL:

- 1-Web pl, 12" $\times$ $\frac{3}{8}$ " $\times$ 1'-11"
- 2-Seat L $\frac{3}{4}$ " $\times$ 6" $\times$ 3 $\frac{1}{2}$ " $\times$ $\frac{3}{8}$ " $\times$ 1'-0"
- 2-Connection L $\frac{3}{4}$ " $\times$ 4" $\times$ 3 $\frac{1}{2}$ " $\times$ $\frac{3}{8}$ " $\times$ 1'-0"
- 1-Stiffener L $\frac{3}{4}$ " $\times$ 2 $\frac{1}{2}$ " $\times$ 2" $\times$ $\frac{3}{8}$ " $\times$ 1'-6"

FIG. 12—TYPICAL BRACKET FOR HIGHLY ECCENTRIC LOAD

RIVETING OF VERTICAL ANGLES TO WEB PLATE

Assume 8 rivets through the vertical connection angles and the web plate, as shown in Fig. 12. This line of rivets must resist 24,000 lb. applied parallel to it and at a distance of 7 in., as shown in Fig. 13(a), that is a direct force of 24,000 lb. and a moment of $24,000 \times 7 = 168,000$ in.-lb.

Direct force D , on each rivet $= 24,000/8 = 3,000$ lb.

Turning force on extreme rivet No. 1, distant d_1 from the centre of gravity of the group $= T_1 = Md_1/\sum d^2 = 168,000 \times 9.75 / \{ 2 \times (1.5)^2 + 2 \times (4.5)^2 + 2 \times (7.5)^2 + 2 \times (9.75)^2 \} = 168,000 \times 9.75 / 348 = 4,700$ lb.

Combining the vertical force of 3,000 lb. with the horizontal force of 4,700 lb., the resultant force on rivet No. 1 (or on No. 8) is found to be $\{ (3,000)^2 + (4,700)^2 \}^{1/2} = 5,600$ lb. As the safe stress on a rivet in this situation is 5,630 lb., the extreme rivets are safe and eight rivets are sufficient.

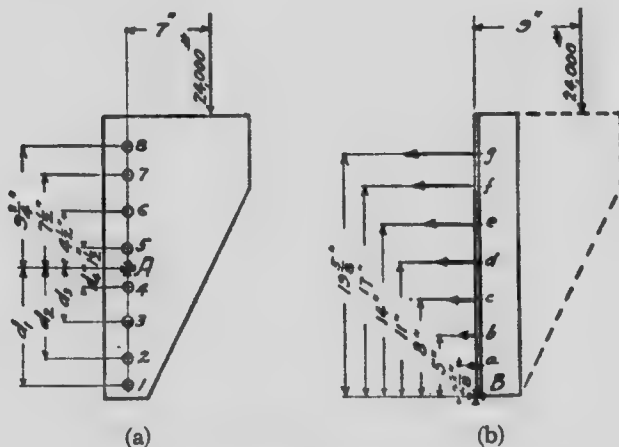


FIG. 13—TURNING FORCES ON RIVETS

RIVETING OF VERTICAL ANGLES TO COLUMN

The rivets connecting the bracket to the column must be sufficient to resist a vertical downward force of 24,000 lb. and a moment of $24,000 \times 9 = 216,000$ in.-lb., assuming the tendency to turn to be about the point "B," Fig. 13 (b).

If seven pairs of rivets are assumed, staggering with the rivets through the web plate, the direct shearing stress on each rivet is $24,000/14 = 1,720$ lb. Since the least value of a rivet in this situation is its single shearing value or $0.442 \times 10,000 = 4420$ lb., the fourteen rivets are adequate for the shearing effect.

The turning moment imposes a tension on each of the upper pair of rivets of $Md_s / \sum d^2 = 216,000 \times 19.625 / \{2 \times (2.375)^2 + 2 \times (5)^2 + 2 \times (8)^2 + 2 \times (11)^2 + 2 \times (14)^2 + 2 \times (17)^2 + 2 \times (19.625)^2\} = 216,000 \times 19.625 / 2171 = 1950$ lb. As the safe tensile force on one rivet is $0.442 \times 7,000 = 3,100$ lb., the 14 rivets are adequate.

The length of the vertical angles is determined by the number and spacing of the rivets through the web plate.

STIFFENER ANGLES

Length of unsupported outer edge of web plate = 19 in., approximately. Ratio of unsupported length to thickness = $19/0.375 = 51$. Hence, stiffeners are necessary as this exceeds 30. Use two angles, $2\frac{1}{2} \times 2 \times 5/16$ in., with 3 rivets, as shown in Fig. 12.

DESIGN OF A QUEEN-POST TRUSSED BEAM

DATA

Span. 30 ft. centre to centre of end bearings, divided into three panels of 10 ft. each.

Depth. Such that the slope of the end section of tie rod is 1 vertical to 4 horizontal.

Load. 1710 lb. per lineal foot superimposed, plus weight of beam, which will be assumed as 90 lb. per lineal foot.

Materials.

Timber to be long-leaf yellow pine weighing $4\frac{1}{2}$ lb. per ft. B.M.,

Tie rods, of soft steel,

Washers, mild steel plates,

Struts, of cast iron,

End supports, of crushed stone concrete.

Permissible Stresses.

Long-leaf yellow pine—

- Compression, endwise of grain, 1,500 lb. per sq. in.,
- Compression, across grain, 350 lb. per sq. in.,
- Bending, 1,500 lb. per sq. in.,
- Combined bending and compression, 1,500 lb. per sq. in.
- Shearing across grain, 1,500 lb. per sq. in.,
- Shearing parallel to grain, 200 lb. per sq. in.

Soft steel—

- Tension, 15,000 lb. per sq. in.

Cast Iron—

- Compression, on short specimens, 16,000 lb. per sq. in.

Assumptions. Consider the beam as sufficiently well supported laterally to render unnecessary the reduction of the allowable bending and compressive stresses to compensate for buckling.

Assume the supporting piers as sufficiently stable in themselves not to require a rigid connection of the beams to them.

TOP CHORD

Section for Shear

Maximum shear, V , in top chord, occurs at the two struts or posts and on the sides nearest the end supports.

$$\begin{aligned} V &= 6/10 wp \\ &= 6/10 \times 1800 \times 10 = 10,800 \text{ lb.} \end{aligned}$$

Required area for vertical shearing stress, allowing for the fact that the intensity of shearing stress at the neutral axis of a rectangular section is $3/2$ the average stress, is $3/2 \times 10,800/1,500 = 10.8$ sq. in.

Required area for horizontal shearing stress = $3/2 \times 10,800/200 = 81$ sq. in.

As will be seen later the section will be fixed by the combined bending and compression, and so it is unnecessary to assign a section for shear.

Section for Moment and Axial Compression

Maximum moment in top chord of beam occurs at struts and is $-1/10 wp^2$, or numerically, $-1/10 \times 1,800 \times (10)^2 \times 12 = -216,000$ in.-lb.

Horizontal, or axial, compression in top chord, $H = 11/10 wp \cot a$, where a is the angle of slope of the end section of the tie rod. Numerically, $H = 11/10 \times 1800 \times 10 \times 4 = 79,200$ lb.

Assume 3 pieces of 4 in. $\times$ 12 in. long-leaf yellow pine. Area, $A = 144$ sq. in. Section modulus, $S = 1/6 bd^2 = 1/6 \times 12 \times (12)^2 = 288$.

Maximum direct compressive stress, $f_1 = H/A = 79,200/144 = 550$ lb. per sq. in.

Maximum bending stress, $f_2 = M/S = 216,000/288 = 750$ lb. per sq. in.

Total maximum stress $= f_1 + f_2 = 550 + 750 = 1,300$ lb. per sq. in.

Stress is lower than is permitted (viz. 1500 lb. per sq. in.), but this is the most satisfactory section that can be built up of commercial timber. Dressing would bring the stress up to approximately 1500 lb. per sq. in.

TIE ROD

Tension in tie rod $= T = 11/10 wp \operatorname{cosec} a = 11/10 \times 1800 \times 10 \times 4.12 = 81,600$ lb.

Required area $= 81,600 / 15,000 = 5.44$ sq. in.

Use soft steel round rods, 1-7/8 in. dia. having an area in the body of 5.52 sq. in. Ends will be upset to 2-3/8 in. dia. giving an area at root of thread 24% in excess of area in body of rods. (See Cambria Steel or Carnegie Pocket Companion).

STRUTS

Compression $= 11/10 wp = 11/10 \times 1,800 \times 10 = 19,800$ lb.

Required area in compression $= 19,800 / 16,000 = 1.24$ sq. in.

Since no ribs should be less than 5/8 in. thick, the area provided must be greatly in excess of structural requirements. (See detail, Fig. 14).

BEARING ON SUPPORTS

$$\text{End reaction} = R = 3/2 wp = 3/2 \times 1,800 \times 10 = 27,000 \text{ lb.}$$

Required bearing area, based on safe compression of long-leaf yellow pine across grain = $27,000 / 350 = 77 \text{ sq. in.}$ Required length of bearing of beam on supports therefore is $77 / 12 = 6.4 \text{ in.}$ A length of 8 in. is shown in the detail.

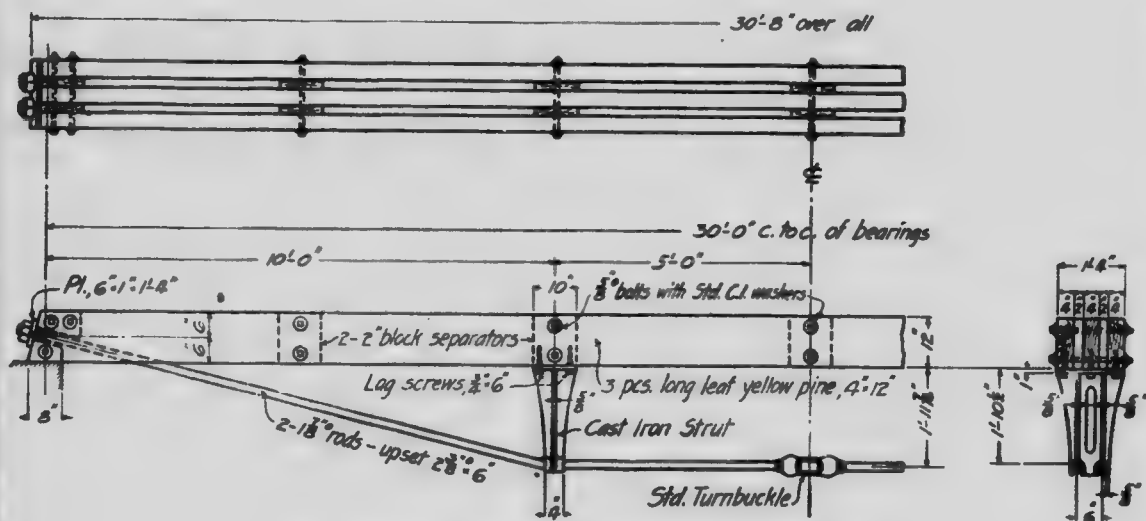


FIG. 14—DETAIL OF 30-FT. QUEEN-POST TRUSSED BEAM

DETAILS

Details of beam are given in Fig. 14.

Washer plates at ends of tie rods must transmit rod tension to ends of top chord. Tension in rod, $T = 81,600 \text{ lb.}$ Required area of plate, governed by end bearing on timber, = $81,600 / 1,500 = 54.4 \text{ sq. in.}$ Use a $6 \times 1 \times 16 \text{ in.}$ steel plate giving area bearing on ends of three $4 \times 12 \text{ in.}$ pieces of 72 sq. in.

Wood block separators bolted as shown, are inserted about 5 ft. centres to stiffen timbers against lateral buckling.

Turnbuckles are inserted at centres of tie rods for the purpose of adjustment.

SAFE CAPACITY OF A BEAM CONNECTION

DATA

Type. Connection for a 12-in I at 31.8 (formerly 31.5) lb., as shown in Fig. 15.

Permissible Stresses.

Shearing on shop rivets = 10,000 lb. per sq. in.,

Bearing on shop rivets = 20,000 lb. per sq. in.

Rivets. $\frac{3}{4}$ in. dia.

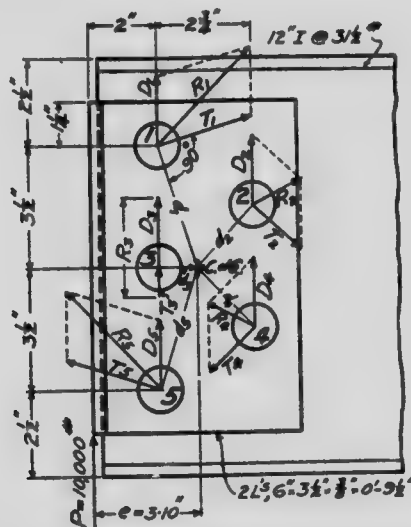


FIG. 15—STUDY OF A BEAM CONNECTION

SOLUTION

Assume a reaction, P , for the beam = 10,000 lb.

Direct force, D , on each rivet = $P \div$ number of rivets = $10,000 / 5 = 2,000$ lb.

Turning moment on connection, $M = Pe = 10,000 \times 3.10 = 31,000$ in.-lb.

Turning force on any rivet = $T_n = Md_n / \Sigma d^2$.

T_n for each rivet is given in accompanying table.

Resultant force on each rivet found by combining D and T_n vectorially as shown in Fig. 15. Resultants thus obtained are given in last column of table.

ANALYSIS OF CONNECTION FOR $P = 10,000$ LB.

Rivet	d in.	d^2 in. ²	Direct Force D lb.	Turning Force T_n lb.	Resultant R_n lb.
1	3.67	13.45	2000	2870	3950
2	2.41	5.80	2000	1880	1520
3	1.10	1.21	2000	860	2860
4	2.41	5.80	2000	1880	1520
5	3.67	13.45	2000	2870	3950

$$\Sigma d^2 = 39.71$$

Safe capacity of connection = assumed reaction $P \times$ least value of rivet + maximum resultant.

Least value of rivet = bearing on 0.35-in. web of beam = $0.75 \times 0.35 \times 20,000 = 5250$ lb.

Maximum resultant (from table) = 3950 lb.

$\therefore$ safe capacity of connection = $10,000 \times 5,250 / 3,950 = 13,300$ lb.

DESIGN OF A STEEL BOX GIRDER

DATA

Span. 22 ft. clear. Assuming distance from face of support to centre of bearing as 8 in., centre to centre span = 23 ft. 4 in.

Form of Section. Girder will be composed of two I-beams not over 20 in. deep with cover plates not over 14 in. wide.

Loads. Superimposed load = 10,000 lb. per lineal ft. uniformly distributed.

Weight of girder (assumed) = 270 lb. per lineal ft.

Total load = 10,270 lb per lineal ft.

Permissible Stresses.

Bending = 16,000 lb. per sq. in.,

Shearing on webs of beams = 10,000 lb. per sq. in.,

Shearing on shop rivets = 10,000 lb. per sq. in.,

Bearing on shop rivets = 20,000 lb. per sq. in.,

Buckling on beam webs = $f_b = R/(a + d/4)$ t must not exceed $f_b = 19000 - 173 d/t$

where R = end reaction for one beam in lb.,

a = length of bearing in in.,

d = depth of beam in in.,

t = thickness of beam web in in.

Bearing pressure on concrete supports = 400 lb. per sq. in.

Rivets. $\frac{3}{4}$ in. dia.

Assumptions. For simplicity, deduct rivet holes in both compression and tension flanges. Consider holes as 7/8 in. dia.

SECTION FOR BENDING

Maximum Moment.

Uniformly distributed load = 10,270 lb. per lineal foot.

Maximum moment = $Wl/8 = (10,270 \times 23.33 \times 280)/8 = 8,387,000 \text{ in.-lb.}$

Required Section Modulus at Centre

$$S = M / f = 8,387,000 / 16,000 = 524.$$

Section modulus provided

Assume a centre section consisting of

2 I's, 20 in. at 65.4 lb.

2 pls., 14 × ¾ in.

2 pls., 14 × 5/8 in.

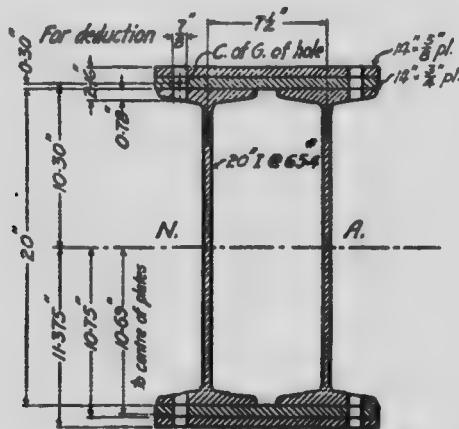


FIG. 16—SECTION OF BOX GIRDER

arranged as shown in Fig. 16. This will be reduced at the proper point by cutting off the outside $\frac{3}{8}$ -in. cover plates.

Gross moment of inertia of two 20-in. I's at 65.4 lb. about neutral axis of girder = $2 \times 1169.5 = 2339 \text{ in.}^4$.

Gross moment of inertia of four flange plates = $2(I_1 + Ay_1^2)$ where

I_1 = moment of inertia of plates on one flange about their gravity axis,

A = area of plates on one flange,

y_1 = distance of gravity axis of plates on one flange from neutral axis of girder.

Since I_1 is negligible compared with Ay_1^2 the moment of inertia of the four plates may be written approximately as $2Ay_1^2 = 2 \times (14 \times 1.375) \times (10.69)^2 = 4400 \text{ in.}^4$.

Hence total gross moment of inertia of section = $2339 + 4400 = 6739 \text{ in.}^4$.

To obtain net moment of inertia, deduct moment of inertia of four $\frac{3}{8}$ -in. holes through 2.16 in. of metal, that is approximately, $4 \times (0.875 \times 2.16) \times (10.30)^2 = 803 \text{ in.}^4$. Net moment of inertia of section then = $6739 - 803 = 5936 \text{ in.}^4$.

Net section modulus = Net moment of inertia divided by distance from neutral axis to extreme fibre = $5936/11.375 = 521.5$. This is sufficiently near the requirement, 524.

Since the moment decreases rapidly near the ends of the girder, the outer cover plates will be required for only a fraction of the girder length. The inner cover plates will be carried full length. For the section where inner cover plates only are employed, the net moment of inertia 5936 will be diminished by the net moment of inertia of the outer plates, that is by approximately, $2 \times \{ (14 - 1.75) \times 0.625 \} \times (11.06)^2 = 1875 \text{ in.}^4$. The net moment of inertia of the beams and the two inner plates is therefore $5936 - 1875 = 4061 \text{ in.}^4$ and the net section modulus = $4061/10.75 = 378$.

The theoretical point of cut-off of the outer $\frac{3}{8}$ -in. plates will be determined graphically from Fig. 17. As the loading is uniform, the moment varies as ordinates to a parabola with vertex at mid-span and axis vertical, and the section modulus will vary likewise. From the figure it is seen that the outer plates might theoretically be cut off at points 6 ft. 2 in. from the centre, but are carried on 9 in. farther at each end to engage two extra lines of rivets, so that they are capable of taking stress at the point where it first needs to be taken.

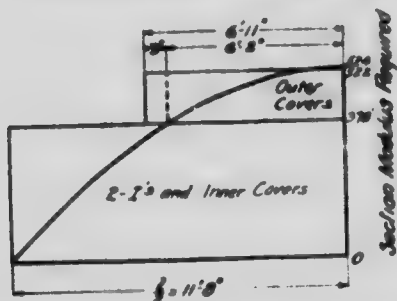


FIG. 17—DETERMINATION OF LENGTH OF COVER PLATES

SECTION FOR SHEAR

End reaction = $10,270 \times 23.33/2 = 119,800$ lb.

Required section in beam webs =

$$119,800 / 10,000 = 11.98 \text{ sq. in.}$$

Section provided = $2 \times 20 \times 0.5 = 20$ sq. in., and is therefore sufficient.

SECTION FOR BUCKLING

Existing buckling stress on web of one beam = $f_b = R / (a + d/4) t = 119,800 / 2 (14 + 20/4) 0.5 = 6,300$ lb. per sq. in., assuming the bearing as 14 in.

Permissible buckling stress on web = $p_b = 19,000 - 173 d/t = 19,000 - 173 \times 20/0.5 = 12,100$ lb. per sq. in.

Hence section is safe for web buckling.

BEARING ON SUPPORTS

End reaction = 119,800 lb.

Required bearing area on concrete support = $119,800 / 400 = 299.5$ sq. in.

Use a 14×22 in. bearing plate = 308 sq. in.

Thickness of bearing plate must be sufficient to transmit pressure of 400 lb. per sq. in. to extreme edges of plate without exceeding bending stress of 16,000 lb. per sq. in. on plate.

Bearing plate acts as cantilever beam, Fig. 18, subjected to upward load of 400 lb. per sq. in.

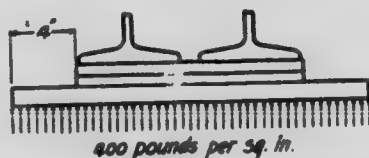


FIG. 18—CANTILEVER BEAM ACTION OF BEARING PLATE

Moment at edge of plate on strip 1 in wide = $\frac{1}{2} \times 400 \times (4)^2$
 = 3,200 in.-lb.

Section modulus S required = $3,200 / 16,000 = 0.2$.

But $S = 1/6 bt^3$, where b is breadth and t thickness of rectangular section. Hence

$1/6 bt^3 = 0.2$ and $t = 1.09$ in.

Adopt a thickness of $1\frac{1}{8}$ in.

DETAILS

For details, see Fig. 19.

Number of rivets required in each flange from centre of bearing to centre line of girder is the number necessary to transmit the total stress in the two plates from the beams into those plates.

Average fibre stress in flange plates
 = $16,000 \times 10.69 / 11.375 = 15,050$ lb. per sq. in.

Total stress in two plates = net area $\times 15,050$.
 = $16.85 \times 15,050 = 253,700$ lb.

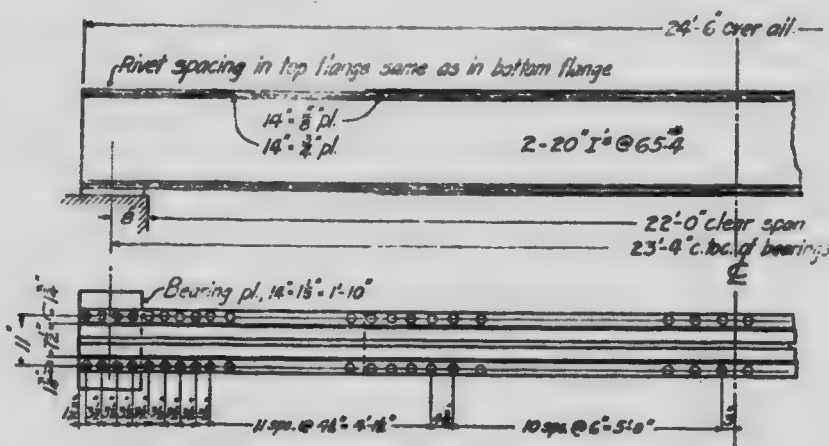


FIG. 19—DETAIL OF 22-FT. BOX GIRDER

Least value of a $\frac{3}{4}$ -in. rivet in this situation is its value in single shear = $0.442 \times 10,000 = 4,420$ lb.

Number of rivets required from centre of bearing to centre line of girder = $253,700 / 4,420 = 57$, or say 29 in each gauge line.

Since maximum allowable spacing is 6 in., the rivets are spaced at 6 in. centres from the centre to a point 5 ft. 3 in. from the centre of the girder, beyond which the spacing diminishes gradually to $3\frac{1}{2}$ in. to allow for the rapid increment in flange stress near the ends.

The rivets connecting bearing plate to girder must be countersunk on under side.

DESIGN OF A PLATE GIRDER

DATA

Span. 40 ft. centre to centre of bearings.

Loads. Superimposed uniformly distributed static load = 6,000 lb. per lineal ft.

Weight of girder (assumed) = 210 lb. per lineal ft.

Total load = 6,210 lb. per lineal ft.

Permissible stresses.

Bending, tension flange = 16,000 lb. per sq. in.

Bending, compression flange = 19,000 — $300\ l/b$, but not over 16,000 lb. per sq. in. (l = unsupported length, b = width of flange).

Shearing on web plate = 10,000 lb. per sq. in. gross area, (Uniformly distributed).

Shearing on shop rivets = 12,000 lb. per sq. in.,

Bearing on shop rivets = 24,000 lb. per sq. in.

Bearing pressure on concrete supports
= 600 lb. per sq. in.

Rivets. $\frac{3}{4}$ in. dia.

Assumptions. Consider one-eighth of the gross area of the web as effective flange area. Assume that top flange is supported at intervals not exceeding 12 times its width. Web thickness not to be less than $1/160$ clear vertical distance between flange angles. Minimum thickness of material, $5/16$ in.

SECTION FOR SHEAR

Maximum end shear.

Total uniformly distributed load = 6210 lb. per lineal foot.

End shear = $6210 \times 40 \times \frac{1}{2} = 124,200$ lb.

Required Web Section

$$124,200 / 10,000 = 12.42 \text{ sq. in., gross.}$$

Web section provided.

Adopting a depth for girder of one-tenth the span, a $48 \times 5/16$ -in. web plate (gross area = 15.0 sq. in.) will suffice for shear. A $5/16$ -in. web is thicker than $1/160$ of clear distance between flange angles. Closeness of flange rivet spacing at ends may necessitate thickening of web to increase bearing area of rivets.

SECTION FOR MOMENT

Maximum Moment (at centre)

$$\begin{aligned} M &= \frac{1}{8} w l^2 = \frac{1}{8} \times 6210 \times (40)^2 = 1,242,000 \text{ ft.-lb.} \\ &= 1,242,000 \times 12 = 14,904,000 \text{ in.-lb.} \end{aligned}$$

Required Tension Flange Section

$$A_f + \frac{1}{8} A_w = M/fd = 14,904,000/16,000 \times 48 = 19.45 \text{ sq. in.,}$$

d being assumed as equal to the depth of the web plate.

Tension Flange Section Provided

$$\begin{aligned} 1/8 \text{ of web} &= \frac{1}{8} \times 48 \times 5/16 = 1.88 \text{ sq. in.} \\ 2 \text{ angles, } 6 \times 4 \times \frac{3}{8} \text{ in.; less four } \frac{7}{8}\text{-in. holes} &= 9.53 \text{ sq. in. net.} \\ 2 \text{ plates } 13 \times \frac{3}{8} \text{ in.; less four } \frac{7}{8}\text{-in. holes} &= 8.44 \text{ sq. in. net.} \\ &= 19.85 \text{ sq. in. net.} \end{aligned}$$

The section chosen conforms to the customary requirement that at least 50% of the flange area, in excess of the web equivalent, should be made up of angles.

Required Compression Flange Section

Since unsupported length of flange is 12 ft. and width is 13 in., $l/b = 12 \times 12/13 = 11$. Hence permissible stress is $p = 19,000 - 300 \times 11 = 15,700$ lb. per sq. in. Required gross area of flange = $14,904,000/15,700 \times 48 = 19.80$ sq. in. As the net area of the tension flange is greater than this, the top flange will be made same as the bottom.

LENGTH OF COVER PLATES

Length of outer cover plate = $x_1 = l(A_1/A)^{1/4}$, where l = length of span, centre to centre, A_1 = area of outer cover, and A = total

net flange area, including web equivalent. For the present case, $x_1 = 40 (4.22/19.85)^{1/2} = 40 \times 0.462 = 18.48$ ft. Adding about 9 in. at each end to provide for at least two lines of rivets, the adopted length becomes 20 ft.

Length of second cover plate, from the outside, $= x_2 = l \{ (A_1 + A_2) / A \}^{1/2} = 40 (8.44 / 19.85)^{1/2} = 40 \times 0.652 = 26.08$ ft. Adding about 18 in., adopted length becomes 27.5 ft. As this is a building girder, it is not necessary to protect the upper flange by carrying the inner cover the full length of the girder.

END STIFFENERS AND BEARING PLATES

Total end reaction = 124,200 lb.

Assuming permissible compression on end stiffener angles as 15,000 lb. per sq. in., area of angles required $= 124,200 / 15,000 = 8.32$ sq. in.

Use four angles, $5 \times 3 \times 5/16$ in. $= 9.60$ sq. in., with long legs turned out and arranged as shown in Fig. 20. Fillers are used under angles.

Number of rivets required in two pairs of angles = end reaction divided by bearing value of one rivet on 5/16-in. web $= 124,200 / 5,630 = 22$. Eleven rivets are used in each pair exclusive of rivets in flange angles.

Required area of bearing plates = end reaction divided by permissible bearing pressure on concrete $= 124,200 / 600 = 208$ sq. in. In order to secure close rivet spacing in the flanges at the ends, use a plate 13×18 in. $= 234$ sq. in.

Net thickness of bearing plate after planing should be at least $t = l (3p/f)^{1/2}$, where l = overhang of plate in inches, p = actual upward pressure on plate in lb. per sq. in., and f = permissible bending stress on plate. Since the overhang, l , is 2.85 in., and p = 530 lb. per sq. in., $t = 2.85 (3 \times 530 / 16,000)^{1/2} = 0.90$ in. To allow for planing, a plate 1 in. thick will be provided.

INTERMEDIATE STIFFENERS

Applying the rule that the spacing of stiffeners should not exceed $s = 64 t (16,000/v)$, where t = thickness of web, and v = intensity of shearing stress in lb. per sq. in. on the web at the section considered, the safe spacing at a point 2 ft. from the centre of the girder bearing is found to be 43 in. For a section 6 ft. from the end the spacing is found to be 55 in. However, practical considerations make a spacing greater than the depth of web undesirable. A spacing of 4 ft. each way from the centre line will therefore be adopted as shown in Fig. 20.

The outstanding legs should be about $1/30$ depth of girder + 2 in. = $48/30 + 2 = 3.6$, say $3\frac{1}{2}$ in. Use two $3\frac{1}{2} \times 3 \times 5/16$ -in. angles, crimped, except at the web splice where $5/16$ -in. fillers are needed to permit the spacing of rivets in stiffeners near the ends. For other intermediate stiffeners, rivet spacing may be 5 in.

FLANGE RIVETING

It will be sufficient to compute the rivet spacing at the centre of each panel. For the bottom, or unloaded flange, the pitch, p , = rh^1/KV , while for the top, or loaded flange, $p = r / \{(KV/h^1)^2 + w^2\}^{1/2}$, where r = least value of rivet, h^1 = vertical distance between gauge lines in vertical legs of flange angles, K = ratio of net area of angles and covers in flange at section considered to total net flange area at section, including one-eighth of web, V = total vertical shear at the section, and w = load directly applied to loaded flange, in lb. per lineal inch.

Pitch at Support.

Least value, r , of a $3/4$ -in. rivet is bearing on $5/16$ -in. web = $3/4 \times 5/16 \times 24,000 = 5,630$ lb.

$$h^1 = 48.5 - 2 \times 2.5 = 43.5 \text{ in.}$$

K = Area of two $4 \times 4 \times 5/8$ -in. angles, with four $7/8$ -in. holes deducted ÷ (net area of the two angles + $1/8$ of gross area of web)
This = $9.53/(9.53 + 1.88) = 0.83$.

$$V = 124,200 \text{ lb.}$$

$$w = 6000/12 = 500 \text{ lb. per lineal inch.}$$

$$\text{Hence, pitch for bottom flange} = p = 5630 \times 43.5 / 0.83 \times 124,200 = 2.37 \text{ in.}$$

$$\text{For top flange, } p = 5,630 / \{(0.83 \times 124,200 / 43.5)^2 + (500)^2\}^{1/2} = 2.33 \text{ in.}$$

A pitch of $2\frac{3}{8}$ in. in each flange is adopted to facilitate the details, as shown in Fig. 20.

As the spacing arrived at is not prohibitively small, the 5/16-in. web plate does not require to be thickened.

Spacing at 2 ft. from end.

$$\text{Here, } V = 124,200 - 6,210 \times 2 = 111,800 \text{ lb.}$$

r , K , h^1 and w same as for end.

p , for bottom flange = 2.63 in.

p , for top flange = 2.56 in.

Adopt a spacing of $2\frac{1}{2}$ in. for both flanges.

Spacing at 6 ft. from end.

$$V = 124,200 - 6,210 \times 6 = 86,900 \text{ lb.}$$

p , for bottom flange = 3.37 in.

p , for top flange = 3.23 in.

Adopt a spacing of $3\frac{1}{4}$ in. for both flanges in this panel.

Spacing at 10 ft. from end.

$$V = 124,200 - 6,210 \times 10 = 62,100 \text{ lb.}$$

K = Net area of two angles and one cover plate. $\div$ total net area of one flange at section.

Deducting four 7/8-in. holes from the two angles and two holes from the cover plate, this = $(9.53 + 4.22)/(9.53 + 4.22 + 1.88) = 0.88$.

p , for bottom flange, = 4.47 in.

p , for top flange = 4.14 in.

Adopt a spacing of 4 in. for both flanges.

Spacing at 14 ft. from end.

$$V = 124,200 - 14 \times 6,210 = 37,300 \text{ lb.}$$

K = Net area of two angles and two cover plates $\div$ Total net area of one flange at section = $(9.53 + 8.44)/(9.53 + 8.44 + 1.88) = 0.91$.

p , for bottom flange = 7.2 in.

p for top flange = 6.1 in.

Since 6 in. is maximum allowable spacing, adopt this for both flanges,

Spacing at 18 ft. from end

Adopt 6 in.

RIVETING OF COVER PLATES

Pitch of rivets in cover plates, if rivets are in pairs, opposite to each other $= p = 2 r^1 h^1 / k^1 V$. Spacing will be computed at theoretical ends of each cover plate, that is at points 7 and 10.7 ft. from centre of end bearings, for the inner and outer covers respectively.

Pitch at Theoretical End of Inner Plate

r^1 = single shearing value of $\frac{3}{4}$ -in. rivet = 5,300 lb.

h^1 = 43.5 in.

K^1 = Net area of one cover plate $\div$ Total net flange area at section = $4.22 / (9.53 + 4.22 + 1.88) = 0.27$.

$V = 124,200 - 6210 \times 7.0 = 80,700$ lb.

Hence, $p = (2 \times 5300 \times 43.5) / (0.27 \times 80,700) = 21.2$ in.

Actual spacing in the line of stress must not exceed 6 in. At the ends of cover plates it should be comparatively close for a couple of feet at least. In this case it is made 4 in. (see Fig. 20).

Pitch at Theoretical End of Outer Plate

r^1 and h^1 are as before.

K^1 = Net area of two cover plates $\div$ Total net flange area at section = $8.44 / (9.53 + 8.44 + 1.88) = 0.43$.

$V = 124,200 - 6210 \times 10.7 = 57,700$ lb.

Hence, $p = (2 \times 5,300 \times 43.5) / (0.43 \times 57,700) = 18.6$ in.

Actual spacing at ends is made 5 in., as shown in Fig. 20.

Pitch at Other Points

Except at the ends of cover plates, the pitch is made the maximum permissible, that is 6 in.

WEB SPLICE

Shear at centre of girder is zero, hence web is spliced only for bending.

Moment of resistance of web = $1/8 f d A_w$
 $= 1/8 \times 16,000 \times 48 \times 48 \times 5/16 = 1,440,000$ in.-lb.

Use two vertical splice plates $12 \times 5/16$ in. and two horizontal splice plates 3 in. wide on flange angles.

Assume arrangement of rivets in vertical splice plates shown in Fig. 21. Since safe stress on flange rivets, 21.75 in. from the neutral axis is 5,630 lb., safe stress on a rivet 1 in. from neutral axis is $5,630 / 21.75 = 260$ lb.

5 2/6 8.

9W
12 1/16

$$\begin{array}{r} 2 \overline{) 7.16} \\ \underline{4} \\ 3 \end{array}$$

1/4 AW

$$\begin{array}{r} 112 \\ 2 \overline{) 224} \\ \underline{448} \\ 448 \\ \underline{448} \\ 0 \end{array}$$

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